5.13 Drainage

5.13.1 Introduction

The Drainage Section of the EIR analyzes the potential short-term, long-term, and cumulative impacts resulting from the construction and operation of the Project and alternatives. The drainage discussion will analyze the drainage conditions in the proposed Shingle Springs Interchange region.

5.13.2 Environmental Setting

Climate

The winters are cool and moist with hot dry summers. The peak rainfall period is between January and March when the jet stream tends to dip south of San Francisco Bay and the storm track takes a southwesterly route from the Pacific through the Bay Area and into the Sacramento-San Joaquin Delta. Rainfall increases rapidly with elevation as the clouds pile up against the Sierra Nevada Mountains. Within twenty (20) miles (32.2 km) one can experience fifteen (15) inches to forty (40) inches (0.38 m to 1.0 m) of rain per year. This rainfall pattern is predictable with storm durations of 12- 24 hours. Within these total storm durations, there can be periods of more intense short duration rainfall, particularly as the main cold front approaches. Summer precipitation is rare, usually caused by monsoonal moisture from either the Gulf of Mexico or Gulf of California. While these summer monsoons may bring thunderstorms to the higher elevations along the crest of the Sierras, the lower elevations may receive nothing more than a sprinkling.

Precipitation in the Slate Creek watershed of Western El Dorado County averages between thirty (30) inches and thirty-six (36) inches (0.8 m and 0.9 m) annually (NOAA, 1973). The nearest weather station is in Placerville located seven (7) miles (11.3 km) east of the project site. Rainfall data from Placerville was not used because the annual average is between thirty-seven (37) and thirty-eight (38) inches (0.9 m - 0.96 m), while the average for the entire Slate Creek Watershed is approximately thirty-five (35) inches (0.88 m). Therefore, the NOAA isopluvials, lines of equivalent annual precipitation, were used to interpolate the annual precipitation, which is estimated to be thirty-four (34) inches (0.86 m) accounting for elevation and historic rainfall isopluvials.

Watershed Setting

The Proposed Project is located within the Slate Creek Watershed, which is approximately 5,365 acres (2,171 ha) in size (**Figure 5.13-1)**. It is identified as a perennial stream on the USGS Shingle Springs Quadrangle. The creek begins in Section 26 of Township 10 North,

FIGURE 5.13-1

Range 10 East on the Shingle Springs Quadrangle and flows west then north under State Highway 50, one-quarter (¼) mile east of the project site. The stream passes through the Grassy Run residential community before merging with Dry Creek approximately five (5) miles (8.0 km) from its headwaters. Land use within the Slate Creek watershed is primarily rural residential with lots varying in size but usually comprising five (5) acres (2 ha) or more.

A small portion of the project site drains to Tennessee Creek, approximately one (1) mile (1.6 km) to the west. Tennessee Creek, is an intermittent stream as identified on the USGS Shingle Springs 7.5 minute Quadrangle. This creek begins in Section 32 of Township 10 North, Range 10 East on the Shingle Springs Quadrangle. A tributary of Dry Creek, it has the same land use characteristics as The Slate Creek Watershed.

Existing Drainage, Culverts, and Bridges

Sub-Basin D1:

Sub-basin D1 is 36.05 acres (14.5 ha) in size **(Figure 5.13-2)**, originating on the north side of Highway 50 at the foot of a cut bank. Water is dissipated by an accumulation of woody debris, detritus, grasses, poison oak, and live oak where sheet flow exits via an asphalt-paved drainage way. Highway runoff joins overland flow from surrounding terrain at the base of the fill slope, which comprises the west-bound emergency turnout of Highway 50. The combined runoff flows in an easterly direction for approximately 1,500 feet (457 m), toward Culvert 1 **(Figure 5.13-1)**, where it is augmented by discharge from an open concrete drain. The cumulative runoff from sub-basin D1 flows to Culvert 1, a 36 inch (0.9 m) diameter corrugated metal culvert. The channel at the culvert inlet is protected from excessive erosion by a concrete apron extending approximately four (4) feet upstream. Based on visual inspection, the maximum headwater depth for Culvert 1 is approximately three (3) meters. **Figure 5.13-2** shows existing drainage patterns within Sub-basin D1.

Sub-Basin D2:

Sub-basin D2 comprises 9.23 acres (3.7 ha) on the north side of Highway 50. Sub-basin D2 runoff consists of overland flow from rural residential property and the southeast corner of the Rancheria. Surface runoff from the section of Highway 50 between Culverts 1 and 2 flows into Sub-basin D2 through an open concrete drain. Sub-basin D2 drains to Culvert 2 **(Figure 5.13-1)**, a twenty-four (24) inch (0.6 m) diameter corrugated metal pipe. Based on visual inspection, the maximum headwater depth for Culvert 2 is approximately two (2) meters. **Figure 5.13-2** shows existing drainage patterns within Sub-basin D2.

FIGURE 5.13-2

Sub-Basin D3:

Sub-basin D3 is 32.23 acres (13.04 ha) in size, located south of the project area, originating at the crest of the Highway 50 cut **(Figure 5.13-2)**. Surface runoff flows approximately twelve hundred (1,200) feet (365.7 m) in an easterly direction, and drains off the highway via a paved curb and gutter. The flow path is, however, blocked by debris, resulting in the diversion of runoff into an open field and creation of an eroded gully on private property adjacent to the highway ROW. Highway runoff combines with overland flow, and discharge from Culverts 1 and 2, at the foot of the east-bound Highway 50 fill slope. Sub-basin D3 has no culvert outlet. The combined runoff from Sub-basins D1 - D3 is tributary to Slate Creek approximately 0.25 mile (0.4 km) east of the project site, and three hundred (300) feet (91.4 m) upstream from the Highway 50 Slate Creek culvert. **Figure 5.13-3** shows existing drainage patterns within Sub-basin D3.

Sub-Basin SC1:

Sub-basin SC1 encompasses the watershed area, excluding sub-basins D1 - D3, tributary to the Highway 50 Slate Creek Culvert. Sub-basin SC1 drains an area of approximately 4,735 acres (1,916 ha). The Highway 50 Slate Creek culvert **(Figure 5.13-1)** is approximately fifteen (15) feet (4.6 m) in diameter, three hundred (300) feet (91 m) in length, and follows the historic channel alignment. Field observations indicate the possible presence of a natural spring in the channel, upstream of the culvert. The culvert outfall has a well-established pool and riparian habitat.

Sub-Basin D4:

Sub-basin D4 **(Figure 5.13-2)** is 52.32 acres (2,117 ha) in extent, draining the southwest portion of the project site, and approximately two hundred (200) feet (60.9 m) of east-bound Highway 50. Roadway runoff is directed via curb and gutter to a paved open drain that flows in a westerly direction, and discharges to a grassy swale at the base of the east-bound Highway 50 fill slope. The swale also collects runoff from the remainder of the sub-basin, and conveys it to Culvert 3 **(Figure 5.13-1)**, a thirty-six (36) inch (0.9 m) diameter corrugated metal pipe. Based on visual inspection, the maximum headwater depth for Culvert 3 is approximately three (3) meters. Sub-basin D4 is tributary to Tennessee Creek. **Figure 5.13-4** shows existing drainage patterns within Sub-basin D4.

Sub-Basin SC2:

Sub-basin SC2 **(Figure 5.13-2)** is comprised of 552.17 acres, representing the incremental drainage area between the Highway 50 Slate Creek culvert, and the confluence of Slate

Figure 5.13-3

FIGURE 5.13-4

Creek with Dry Creek. A one hundred forty-two (142) acre (57.5 ha) portion of sub-basin SC2, hereinafter referred to as sub-basin SC2', is tributary to Slate Creek at the Reservation Road bridge crossing.

The Reservation Road bridge has a span of approximately thirty-five (35) feet (10.6 m), with its low chord approximately eight (8) feet (2.4 m) above the flow line of Slate Creek. Visual inspection of the over bank area indicated high water marks at approximately 8 foot (2.4 m) depth.

Methodology

Watershed Delineation

The majority of the Proposed Project site, approximately ninety-five percent (95%), is within the Slate Creek watershed. The portion of the Project within the Tennessee Creek watershed has previously been analyzed by Caltrans in conjunction with studies of Highway 50 widening. Since design alternatives for the Proposed Project would have minimal impact on discharge to Tennessee Creek, the present study focuses on the Slate Creek watershed.

The boundaries of the Slate Creek watershed, delineated on the USGS 7.5 minute Shingle Springs Quadrangle, are shown in **Figure 5.13-1**. The highest point in the watershed is at the Logtown historic town site, at elevation 2,012 feet (613 m); the lowest point is at the confluence of Slate Creek with Dry Creek, at elevation 1,160 feet (354 m).

The Slate Creek watershed was divided into five sub-basins, designated Sub-basins D1, D2, D3, SC1, and SC2, representing areas tributary to key points for which stream flows were evaluated. A single sub-basin, Sub-basin D4, tributary to Tennessee Creek, was identified. Site inspection was used to verify effects of infrastructure such as roads, culverts, and other drainage structures, on the placement of sub basin boundary lines. Watershed boundaries are shown on **Figure 5.13-1**. The sub-basins that resulted are described in the following paragraphs. **Table 5.13-1** summarizes the sub-basin descriptions.

Soils

Soils are one of the primary influences on surface runoff rates. Infiltration rates of soils vary widely and are affected by subsurface permeability as well as by surface intake rates. The United States Department of Agriculture, Natural Resources Conservation Service, has classified soils into four (4) Hydrologic Soil Groups, based on the minimum infiltration rates of the soils. Hydrologic soil groups are ranked A to D. Soils in group A have the highest

Table 5.13-1 Summary Description Of Drainage Sub-Basins

Sub-Basin	Area (Acres)	Descriptions
D1	36.06	Area tributary to Culvert 1; located north of Hwy 50, adjacent to hwy; tributary to Slate Creek upstream of Hwy 50
D2	9.23	Area tributary to Culvert 2; located north of Hwy 50, adjacent to hwy; tributary to Slate Creek upstream of Hwy 50
D3	32.23	Located south of Hwy 50, adjacent to hwy; receives outflow from sub-basins D1 & D2; discharges directly into Slate Creek
D4	52.32	Area tributary to Culvert 3; located south of Hwy 50, adjacent to hwy; discharges into Tennessee Creek
SC1	4,375	Area (excluding D1-D3) tributary to Hwy 50 Slate Creek culvert; located south of Hwy 50
SC2	552.17	Incremental area tributary to Slate Creek downstream of the Hwy 50 culvert; primarily located north of Hwy 50; SC2 includes most of the Rancheria & the Grassy Run residential community
SC2'	142	Portion of sub-basin SC2; tributary to Slate Creek above Reservation Road

Source: Gene E. Thorne & Associates, Inc., 2001

rates of infiltration and water transmission, while those in group D have low infiltration rates and high runoff potential.

Surface runoff is also influenced by the extent and type of soil cover, or disturbances to the soil profile. Within the project area, cover conditions vary, and include the following: impervious surfaces, bedrock, graveled drives, dirt roads and drives, paved roads and drives, residences and surrounding outbuildings and yards, managed grazing, and undisturbed natural conditions.

As indicated in the El Dorado Soil Survey, the Proposed Project is underlain by soils having two basic hydrologic characteristics. Diamond Springs sandy loam, Dfd, is shown within a portion of the project. In reality, however, this area consists of exposed bedrock. For purposes of runoff analyses, this is treated as impermeable surface area. Soils of the approximately 5.6 acre (2.3 ha) project site, as well as those adjacent to Highway 50, are classified in hydrologic soil group D. Soils within the rest of the Slate Creek watershed were also determined to be in hydrologic soil group D.

The Soil Conservation Service has developed a relationship between soil type and runoff potential, expressed as a runoff curve number (CN). The major factors that determine CN are the hydrologic soil group, cover type, treatment, hydrologic condition, and antecedent runoff condition. Curve numbers representing average antecedent moisture conditions for pre-development conditions within the sub-basins identified in this study (**Table 5.13-2**).

Table 5.13-2 Curve Number Representing Average Antecedent Moisture Condition For Pre-Development Conditions Within Sub-Basins

Sub-Basin	Curve Number
D1	82
D2	86
D3	85
D4	82
SC1	80
SC2	80

Source: Gene E. Thorne & Associates, Inc., 2001

Post-Development Conditions

Any disturbance of soil profile or changes in soil cover will change the runoff characteristics of a watershed. The interchange project will alter approximately 5.6 acres (2.3 ha) within the project site, and a portion of the ROW for the proposed west-bound on-ramp. Future modifications are located in sub-basin D1 only. Infiltration rates in approximately four 4 acres (1.6 ha) within sub-basin D1 would be affected by the project. These changes involve 2.27 acres (0.9 ha) of new roadway and 1.75 acres (0.7 ha) of disturbed surface area adjacent to the interchange and access road.

Implementation of the Proposed Project would raise the runoff curve number of sub-basin D1 from eighty-two (82) to eighty-five (85), due to creation of additional impervious area within the sub-basin. Runoff curve numbers in all other sub-basins are unchanged for post-development conditions. The post-development condition is the same for either of the interchange design alternatives.

Runoff Computations

Rational Method

Peak runoff expected to occur over sub-basins D1 through D4 was computed by means of the Rational Method. The equation is: Q = C I A, where Q is the rate of surface discharge, in cubic feet per second (cfs).

The runoff coefficient “C” is a dimensionless factor representing the percent of water expected to run off the ground surface during the storm. The coefficient “C” is determined through consideration of topographic relief in the sub-basin, soil infiltration capacity, vegetal cover, and availability of surface storage. Runoff coefficients applicable for the ten (10) year storm were determined from Figure 819.2A and Table 819.2B of the Highway Design Manual and adjusted by a frequency factor of 1.25 for computing one hundred (100) year storm runoff.

The value of "I" represents rainfall intensity, in inches per hour, for rainfall duration equal to the time of concentration computed for the sub-basin under analysis. Values of "I" are obtained from rainfall intensity-duration-frequency tables included in the El Dorado County Drainage Manual. For this project an average annual precipitation of thirty-four (34) inches (0.86 m) was obtained. Design discharge, as computed by the Rational Method has the same probability of occurrence (design frequency) as the frequency of the rainfall used.

In order to select the appropriate rainfall intensity for use in the Rational Method equation, it is necessary to know the time of concentration for the sub-basin in question. Time of concentration is defined as the time it takes for water falling on the most hydraulically distant point in the sub-basin to reach the outlet. Times of concentration may be computed using empirical formulas based on flow lengths, and watershed slopes, or may be estimated from field observations. For sub-basins D1 through D4, times of concentration were estimated from field conditions, taking into account surface roughness, debris accumulation, presence of detention structures, and estimated travel distance and route. These values, shown in **Table 5.13-3**, are comparable to those used in the Drainage Report for the Shingle Springs Rancheria Casino EA (Gene E. Thorne & Associates, Inc., 2001).

Determination of sub-basin areas was described in a preceding section.
The Rational Method was used to compute peak discharge for each sub-basin, under both pre and post development conditions. The Proposed Project would change the runoff characteristics of only sub-basin D1. The effects of the project cause the CN in sub-basin D1 to increase to eighty-five (85). The results of the peak discharge computation are summarized in **Table 5.13-3**.

Hydrograph Method (HEC-1)

Use of the Rational Method for peak flow computation is limited to watershed areas less than three hundred twenty (320) acres (129 ha). However, as part of this study, it was desired to compute runoff at key points with drainage areas in excess of three hundred twenty (320) acres (129 ha). These key points are located on Slate Creek where flow from two or more drainage sub basins is combined. A hydrograph method of estimating design discharge is used for determining the combined rate of runoff from two or more drainage areas that peak at different times. Section 819.6 of the Highway Design Manual cites the US Army Corps of Engineers HEC-1 Flood Hydrograph Package as a commonly used method for hydrograph simulation.

Table 5.13-3 Summary Of Peak Runoff Computations For Sub-Basins D1 Through D4

Pre-Development Peak Runoff									
				10-Year Storm			100-Year Storm		
Drainage Shed	**Area (Acres)**	**Curve Number (CN)**	**Time Of Concen tration (Min.)**	**Runoff Coeff. (C)**	**Rainfall Intensity ('I'; In/hr)**	**Peak Runoff Q=CIA (Cfs)**	**Runoff Coeff. (C)**	**Rainfall Intensity ('I'; In/hr)**	**Peak Runoff Q=CIA (Cfs)**
D1	36.05	82	30	0.58	1.22	**25.5**	0.72	1.73	**44.90**
D2	9.23	86	15	0.60	1.7	**9.41**	0.75	2.42	**16.75**
D3	32.23	85	30	0.60	1.22	**23.59**	0.75	1.73	**41.82**
D4	52.32	82	30	0.58	1.22	**37.02**	0.72	1.73	**65.17**
Post-Development Peak Runoff									
				10-Year Storm			**100-Year Storm**		
Drainage Shed	**Area (Acres)**	**Curve Number (CN)**	**Time Of Concen tration (Min.)**	**Runoff Coeff. (C)**	**Rainfall Intensity ('I'; in/hr)**	**Peak Runoff Q=CIA (cfs)**	**Runoff Coeff. (C)**	**Rainfall Intensity ('I'; in/hr)**	**Peak Runoff Q=CIA (cfs)**
D1	36.05	85	30	0.62	1.22	**27.27**	0.78	1.73	**48.65**
D2	9.23	86	15	0.60	1.7	**9.41**	0.75	2.42	**16.75**
D3	32.23	85	30	0.60	1.22	**23.59**	0.75	1.73	**41.82**
D4	52.32	82	30	0.58	1.22	**37.02**	0.72	1.73	**65.17**

Source: Gene E. Thorne & Associates, Inc., 2001

A HEC-1 flow network was developed in order to compute flows in Slate Creek at the Reservation Road Bridge, and at the confluence of Slate Creek with Dry Creek. For the HEC-1 analyses, Sub basins D1 through D3 were treated as a single shed area, while the portion of Sub basin SC2 that contributes to flow in Slate Creek at the bridge location was treated as a separate sub basin, referred to as Sub-basinSC2'. Sub-basin D4 is not tributary to Slate Creek at the key points identified for analysis. Therefore, Sub-basin Dr is not included in the HEC-1 computations.

Input data requirements for HEC-1 are similar to those for the peak discharge method. However, the HEC-1 model simulates surface runoff response over a period of time, rather than as a single value. Precipitation data represents a storm of given duration, with temporal distribution characteristic of storms affecting the watershed location. For the present analyses, a twenty-four (24) hour, SCS Type 1 storm was simulated. Computations are based on the SCS curve number loss rate and use of an SCS dimensionless unit graph. Sub-basin lag times were taken to be equivalent to 0.6Tc, with time of concentration, Tc, estimated using the equation: $Tc = (11.9L^3/H)^{0.385}$, where L is length of watercourse, in miles, and H is elevation difference within the watershed, in feet. HEC-1 input parameters are summarized in **Table 5.13-4** below.

Table 5.13.4 HEC-1 Input Parameters

Sun-Basin	Area (sq. mi.)	Pre-Development CN	Post-Development CN	LAG (hrs.)
D1 thru D3	0.12	83.7	85.1	0.30
SC1	7.40	80	80	1.1
SC2	0.64	80	80	0.60
SC2'	0.22	80	80	0.50

Source: Gene E. Thorne & Associates, Inc., 2001

Results of the HEC-1 hydrograph analyses are shown in **Table 5.13-5**.

Table 5.13-5 Peak Runoff Hydrograph Computations Using HEC-1

Location	10-YEAR STORM		100-YEAR STORM	
	Pre-Development (cfs)	Post-Development (cfs)	Pre-Development (cfs)	Post-Development (cfs)
Slate Creek at Hwy 50	2541	2542	4392	4392
Slate Creek at Reservation Road Crossing	2593	2594	4483	4484
Slate Creek at confluence with Dry Creek	2752	2752	4765	7466

Source: Gene E. Thorne & Associates, Inc., 2001

Table 5.13-6 summarizes the results of the TR-55 hydrograph computations. It should be noted that peak flows computed in the TR-55 analyses are comparable to the HEC-1 results, and show no impact on flows in Slate Creek resulting from project development.

Table 5.13-6 TR-55 Runoff Summary

Pre-Development Runoff at Key Points in Slate Creek Watershed								
					10 Year Storm		100 Year Storm	
Location	Tributary Sub-basin(s)	Area (mi^2)	Composite CN	SCS Storm Type	24 hr. Precip. (in.)	Computed Runoff (cfs)	24 hr. Precip. (in.)	Computed Runoff (cfs)
Hwy 50 Culvert	SC1+D1-DE	7.52	80	I	4.51	2,659	6.39	4,520
Reservation Rd. Bridge	SC1=D1-D3+SC2'	7.74	80	I	4.51	2,737	6.39	4,652
Dry Creek Confluence	SC1+D1-D3+SC2	8.16	80	I	4.51	2,885	6.39	4,904

Table 5.13-6 (Cont.) Tr-55 Runoff Summary

Post Development Runoff at Key Points in Slate Creek Watershed								
					10 Year Storm		100 Year Storm	
Location	**Tributary Sub-basin(s)**	**Area (mi^2)**	**Composite CN**	**SCS Storm Type**	**24 hr. Precip. (in.)**	**Computed Runoff (cfs)**	**24 hr. Precip. (in.)**	**Computed Runoff (cfs)**
Hwy 50 Culvert	SC1+D1-D3	7.52	80	I	4.51	2,659	6.39	4,520
Reservation Rd. Bridge	SC1+D1-D3+SC2'	7.74	80	I	4.51	2,885	6.39	4,652
Dry Creek Confluence	SC1+D1-D3+SC2	8.16	80	I	4.51	2,885	6.39	4,904

Source: Gene E. Thorne & Associates, Inc., 2001

Predicted Discharge

In order to estimate project impacts, it is necessary to calculate the predicted surface discharge over the project area sub-basins. This was done by comparing the pre and post project results for Drainages 1-4. The Rational Method was used on areas SC1 and SC2 and each segment of both design alternatives. Since no construction will occur in drainages SC1 and SC2, no changes to discharge will occur. The same criteria and assumptions are made for the analysis of post-project discharge:

- Run off increases with storm intensity;
- Time of concentration is not altered by the project;
- Field observations and measurements are comparable to the design parameters of culverts.

The only criterion that has changed is the weighted runoff coefficient. This number has been readapted for each separate sub-basin that will have impermeable surface area added, or an altered hydraulic gradient. Separate sets of numbers are available for both interchange designs considered.

The hydraulic gradients and drainage patterns are best maintained in the Flyover design option. Only surface discharge from the east-bound off-ramp will be redirected to Tennessee Creek. The estimated volume is 0.34 cfs. The Diamond Interchange design directs more flow from the Slate Creek sub-basins to the Tennessee Creek. The redirected water volume from the Diamond Interchange design is 0.52 cfs. The locations of the hydraulic gradient breaks are found in **Figure 5.13-5** and **Figure 5.13-6** and **Tables 5.13-7 and 5.13-8**.

Insert Figure 5.13-5

Insert Figure 5.13-6

Table 5.13-8 shows the predicted discharge from each segment of each alternative and the predicted design discharge for each storm return period. **Table 5.13-8** shows the predicted contribution to each sub-basin for both alternatives. Both tables show discharge is comparable for each alternative.

The additional lane (**Figure 5.13-7**) has been discounted because discharges from this are not expected to change since the surface has near total runoff (~0.97) under current conditions. Surface discharge from the addition of this lane will be transported through the existing centerline drainage ditch. Caltrans, in planning for future expansion of Highway 50 from a four (4) lane to a six (6) lane, has considered the additional surface runoff to be a less than significant impact (Rhoads, pers. comm.). No water will be redirected to a neighboring watershed.

Table 5.13-7 Predicted Surface Discharge (cfs) Of Roadway Segments For The Flyover Design And Diamond Design Interchange Alternatives

Interchange Segment	1 hr Storm Return Interval				
	2.33yr	**10 yr**	**25 yr**	**50 yr**	**100 yr**
Flyover Design					
Eastbound off-Ramp	0.69	1.09	1.20	1.32	1.45
Eastbound on-ramp	0.53	0.78	0.92	1.02	1.11
West bound off-ramp	0.46	0.67	0.80	0.89	0.97
Westbound on ramp	0.41	0.61	0.72	0.80	0.87
Artesia	0.10	0.15	0.17	0.19	0.21
Access Road	0.59	0.87	1.03	1.14	1.25
Auxiliary Lane	0.80	1.17	1.39	1.54	1.68
Design Total Discharge (CFS)	**3.57**	**5.25**	**6.06**	**6.89**	**7.53**
Diamond Design					
Eastbound off-Ramp	0.32	0.47	0.56	0.62	0.67
Eastbound on-ramp	0.38	0.56	0.67	0.74	0.81
West bound off-ramp	0.40	0.58	0.69	0.76	0.83
Westbound on ramp	0.28	0.42	0.50	0.55	0.60
Artesia	0.10	0.15	0.17	0.19	0.21
Access Road	1.05	1.55	1.83	2.03	2.28
Auxiliary lane	0.80	1.17	1.39	1.54	1.68
Design Total Discharge (CFS)	**3.33**	**4.89**	**5.81**	**6.42**	**7.02**

Source: AES, 2001; Mark Thomas & Co., Inc., 2001; El Dorado County, 1995

Not all discharge from the Proposed Project has been considered additional discharge. Discharge from altered hydraulic gradients has been considered as redistributed discharges

Insert Figure 5.13-7

from existing sub-basins. **Table 5.13-8** identifies the post-project discharge and receiving drainage of each design alternative. This would still alter the surface drainage but would not be considered additional discharge. Because much of the Proposed Project will be constructed on already impervious surfaces, discharge from these areas is not considered additional. The only discharge to actually be considered as additional is from the access road leading through the 5.6-acre parcel.

TABLE 5.13-8 Predicted Discharge (Cfs) Into Sub-Basins By Interchange Design Alternatives

Alternative	Interchange Discharge to Sub-basin				
Flyover Design	**Drainage 1**	**Drainage 2**	**Drainage 3**	**Drainage 4**	**Return Period Total**
2.33 year	1.91	0.12	0.53	1.01	3.57
10 year	2.81	0.18	0.77	1.49	5.25
25 year	3.33	0.22	0.92	1.59	6.06
50 year	3.69	0.24	1.01	1.95	6.89
100 year	4.03	0.26	1.11	2.13	7.53
Diamond Design					
2.33 year	1.20	0.21	0.74	1.19	3.34
10 year	1.76	0.31	1.08	1.74	4.89
25 year	2.09	0.37	1.28	2.07	5.81
50 year	2.30	0.41	1.42	2.28	6.41
100 year	2.52	0.45	1.55	2.50	7.02

Source: AES, 2001; Mark Thomas & Co., Inc., 2001; El Dorado County, 1995

5.13.3 Impacts And Mitigation Measures

Significance Criteria

Drainage impacts for this project will be considered significant if the additional flows for predetermined storm events exceed the design capacities of existing structures (four culverts and one bridge downstream of the Proposed Project) or cause surface erosion above background levels defined by the NRCS universal soil loss equations. For stream culverts this would mean the exceedance of the allowable headwater and/or causing excessive erosion of the culvert embankments. As it applies to the design capacity of bridges, the additional discharge must not exceed the expected peak flow surface elevation of waters during a one hundred (100) year event or significantly contribute to an existing exceedance. This limit is one (1) foot below the bottom elevation of the bridge. As with culverts, erosion must not be accelerated at the bridge abutments.

Cross culverts must be able to pass a ten (10) year storm flow at no more than 1 pipe diameter water depth. Water in excess of this limit is to be retained behind the road crossing up to a predetermined elevation for a one hundred (100) year storm event. Additions that cause the ten (10)-year event to exceed one (1) pipe diameter or exceed the one hundred (100) year retention elevation will be considered significant.

Impact/ Mitigation

Impact 5.13-1 Peak Flow

AA Since the No Project/Action Alternative will not result in an increase in impervious surfaces the existing surface discharge predictions will remain the same. ***Therefore, the No Project/Action Alternative is not expected to result in a significant impact to the environment.***

AB, AC The maximum expected additional discharge is during a one hundred (100) year, one (1) hour storm. Half of the additional paved area will be constructed on existing impervious surfaces leading to no net increase of peak discharge from these areas. However, impervious surfaces placed on top of the 5.6 acre (2.3 ha) access ~~trust~~ parcel, the Rancheria, and the northern Caltrans right of way will add 2.27 acres (.92 ha) of impervious surface area and 1.75 acres (.71 ha) of other altered surfaces (slopes, fill areas, graded swales, etc.). The soil is already prone to high discharges during storms (82% during a 2.33 year event accounting for slopes and land cover), so the additional increases as part of the weighted average add to a naturally high discharge. The post project weighted runoff coefficient is eighty-five (85), with the predicted change in discharge being three (3) cfs for the impacted project area during a 2.33-year event. These additional discharges and resulting peak flows will not exceed the design requirements of the existing culverts. ***Therefore, the Flyover Interchange Alternative Design and the Diamond Interchange Alternative Design are not expected to result in a significant impact to the environment.***

Mitigation 5.13-1 Peak Flow

None Required

Impact 5.13-2 Structural Alterations To Existing Surface Drainage Patterns

AA Since the No Project/Action Alternative will not result in an increase in impervious surfaces the existing surface discharge predictions will remain the same. ***Therefore, the No Project/Action Alternative is not expected to result in a significant impact to the environment.***

AB The west-bound off-ramp will likely result in the in filling of the drainage channel for Drainage Area 1 (D1). Presently, the water in this channel is down cutting through the native soils before encountering bedrock near Culvert #1. The down cutting begins on the northeast end of the westbound emergency turnout and continues for approximately three hundred (300) feet (91 m) reaching depths of up to eight (8) feet (2.4 m). The cross-section for the westbound ramp shows a graded slope that would result in this channel being filled. This would result in the existing drainage channel being filled and a new channel being constructed closer to private property. ***Therefore, the Flyover Interchange Design Alternative is expected to result in a significant mitigable impact to the environment.***

AC The Diamond design will alter existing hydraulic gradients. These alterations to the hydraulic gradients will transfer water from the Slate Creek watershed to the Tennessee Creek watershed. The elevated off-ramps and roadways will leave open soil underneath. This soil will have different re-vegetation characteristics than pre-project and will alter the soil moisture and storm discharge budget. If the soil is not in optimal condition to receive precipitation, it will not re-vegetate appropriately, thereby generating additional surface discharge and suspended sediments. ***Therefore, the Diamond Interchange Design Alternative is expected to result in a significant mitigable impact to the environment.***

Mitigation 5.13-2 Structural Alterations To Existing Surface Drainage Patterns

The following mitigation will assure that the proposed project will result is a ***less than significant impact***.

(A) Mitigation for AB includes installing a culvert for the length of the filled in channel.

(B) Mitigation for AC includes re-vegetating with appropriate plants for the conditions created by the raised off-ramps and roadways. Mitigation for the altered hydraulic gradients is addressed by the additional discharge being retained in a detention reservoir on the Rancheria after construction of the Casino/Hotel.

Impact 5.13-3 Impacts To Existing Drainage Structures

AA Since the No Project/Action Alternative will not result in an increase in impervious surfaces the existing surface discharge predictions will remain the same. ***Therefore, the No Project/Action Alternative is not expected to result in a significant impact to the environment.***

AB Increases in peak runoff of ≤1 cfs, representing an increase of much less than one percent (1%), are expected to occur at the Slate Creek Highway 50 culvert or at the Reservation Road bridge during a one hundred (100) year event. These additions will not alter the performance of the two crossings during the design storm. On-site culverts will not be impacted by additional post-project discharges. The only existing culvert that may be impacted by construction of the Flyover design is Culvert 1. Both the outlet and inlet to this culvert appear to be affected by the construction of the east-bound on-ramp and the west-bound off-ramp, respectively. According to the engineered drawings for the Flyover alternative, cutting and filling will take place on this culvert. The upper portion of the open concrete drain at the base of the Highway embankment in Sub-basin D2 will be altered by cutting and filling activities as well. ***Therefore, the Flyover Interchange Design Alternative is expected to result in a significant mitigable impact to the environment.***

AC As with the Flyover interchange design, preliminary drawings show that the placement of the east-bound off-ramp and the west-bound on-ramp may interfere with inlets and outlets of Culverts 1 and 2 through cut and fill activities or the placement of pylons at these features. ***Therefore, the Diamond Interchange Design Alternative is expected to result in a significant mitigable impact to the environment***.

Mitigation 5.13-3 Impacts To Existing Drainage Structures

The following mitigation will assure that the proposed project will result is a ***less than significant impact***.

(A) Mitigation measures for AB and AC. Although project runoff does not increase flow in Slate Creek at the Reservation Road bridge, impacts at this structure could be lessened by retaining additional flows on-site within the Caltrans ROW or on the 5.6 acre (2.3 ha) trust parcel until the Casino/Hotel is constructed. Once completed, the Casino surface drainage network will remove 3.12 acres (1.3 ha) from Sub-basin D1, somewhat reducing the design discharge.

(B) Mitigation measures for AB. Impacts to Culvert 1 can be mitigated by either replacing the culvert or creating a box entrance at the inlet side and extending the outlet past the on-ramp.

(C) Mitigation measures for AB. Impacts to Sub-basin D2 structures can be mitigated by relocating the open concrete drain within the Caltrans ROW.

(D) Mitigation measures for AC. Impacts to culvert inlet and outlets by construction can be mitigated by placing pylons at least thirty (30) feet (9.1 m) away from the culverts or re-engineering the culvert inlet and outlets to fit the structural needs at the project site.

Impact 5.13-4 Cumulative Impacts To Drainage

AA Under the No Project/Action Alternative, the interchange would not be constructed; therefore, no impact upon Drainage would occur on or around the project site. ***The No Project/Action Alternative will not result in a cumulative impact to Drainage.***

AB, AC The only project specific drainage impact identified is related to an increase in impervious surface, that will result in an increase in flows into culverts. The implementation of Drainage mitigation measures will assure that Alternative B and C will not significantly add to the cumulative impact of flows upon culverts. ***Therefore, no Drainage impacts are anticipated to occur as a result of the proposed interchange project.***

Mitigation 5.13-4 Cumulative Impacts

None required.